CHAPTER 20

STRUCTURAL STEEL

20.1 Design

The Department uses the load factor method of design as defined in the **AASHTO Standard Specification for Highway Bridges**. The design procedures in **AASHTO LRFD Bridge Design Specifications** shall be used with the approval of the Bridge Design Engineer.

20.1.1 Type of Steel

- Structural steel shall conform to the AASHTO M270 (ASTM A 709), grades designated in **Table 10.2A of the AASHTO Standard Specifications for Highway Bridges**.
- The use of Grades 36, 50 and 50W is permitted. The use of a higher strength steel or hybrid girders, including the welding procedure shall be subject to the approval of the Chief Transportation Engineer.
- The use of AASHTO M270 (ASTM A709), GRADE 50W, "weathering steel", is subject to the cleaning and painting requirements that are specified in the DDOT standards.
- All structural steel plans shall have the following note shown thereon:

STRUCTURAL STEEL: AASHTO M 270, GRADE ___ (ASTM A709, GRADE___) with Supplementary Requirements for Notch Toughness for all member components marked (T).

- The material for all main load-carrying members of steel bridges subject to tensile stresses shall meet AASHTO requirements for notch toughness.
- Designate the main load carrying member components that are subject to tensile stress. The designation (T) shall be noted on the contract plans.
- The components to be designated (T) shall include flanges, webs, and splice plates of the welded stringers, girders, or rolled beams. The above note and designations shall be verified on the shop plans.

20.1.2 Protective Coatings

The designer must provide for painting of all structural steel, except weathering steel. Normally, Light–Grey paint, Standard Color Chip No. 26408, Federal Standard No. 595, is used on bridges over waterways.

Other colors may be used with the approval of the Bridge Design Engineer on other bridges except bridges over waterway.

Weathering steel may be considered for structures over high traffic volume roadways or railroads, where access for painting or repainting is limited or dangerous. The use of weathering steel will be evaluated on a case-by-case basis and is subject to approval of the Bridge Design Engineer. Refer to **FHWA Publication Forum on Weathering Steel for Highway Structures: Summary Report**. Weathering steel shall not be used in corrosive environments where there is high humidity or high concentrations of chloride. It may be desirable to paint the ends of weathering steel girders near bearing areas under joints. Normally, the length of the painted area is equal to one and one-half the depth of the beam. Where weathering steel is painted, brown Standard Color No. 10076, Federal Standard No. 595A, is used.

20.1.3 Span Type Selection

Simple and continuous stringers are within the range of span types that can be considered for the majority of structures. The choice should be made on the basis of judgment, economy, appearance and serviceability. Redundant type (multiple load path) systems shall always be used. Non-redundant (single load path) systems and use fracture critical members should be avoided. A redundant structure has multiple load paths available to share the loads should a single member fail. Fracture-critical structures are not redundant. A fracture-critical structure is a structure where the failure of a single member or component of a member will cause failure of the span. If a design contains fracture-critical members, these members must be specifically identified on the plans.

The approval shall be obtained prior to the Preliminary Plan submission. Such approval will be subject to the special design, fabrication, and plant inspection provisions of the **AASHTO "Fracture Control Plan"**

Continuous spans are only recommended for structures founded on rock point bearing piles, or unyielding soils. The soil may be considered unyielding if the following conditions are met:

- The bearing capacity is at least 4 ksf.
- The available soil data permits the settlement to be reliably computed.
- The effects of the differential settlement are accounted for in the design of the superstructure.

NOTE: Design differential settlement shall be considered at 1 in. maximum.

Structures containing pin and hanger connections for suspended/cantilever spans should be avoided.

To support the redundancy requirements, the following spacings for steel beams are considered:

- Minimum, 7’-6”
- Desirable, 8’- 6”
- Maximum, 9’- 6”

Where vertical clearance is not a problem, a wider spacing may be justified, on a case-by-case basis, with the approval of the Bridge Design Engineer. The Department does not permit the use of cover plates on rolled beams. Use a minimum flange plate thickness of 5/8 in. and width of 12 in. to reduce warping during fabrication, improve transportation stability, and reduce erection problems.

20.1.4 Economics of Stringer Design

Straight beams and girders are preferred because of simplicity of design and lower fabrication costs. The following types of steel beams and girders are considered:

- Rolled I-beams,
- Welded-plate girders,
- Haunched girders, and

20.1.4.1 Rolled I-Beams

Used for spans up to 90 ft. The advantages of rolled I-beams are:

- Economical use of material for shorter spans,
- Simplicity of construction results in savings, and
- Design is straightforward.

20.1.4.2 Welded Plate Girders

Used for spans greater than 90 ft. The advantages of welded-plate girders are:

- Simpler to design than haunched or box girders,
- Simplicity of construction results in savings over haunched or box girders, and
- Fabrication is easier and can be more automated.

20.1.4.3 Haunched Girders

Used for spans greater than 90 ft., and where:

- Vertical clearance cannot be attained with welded-plate girders, or
- Aesthetics is considered.

Haunched girders may be used for spans greater than 130 ft., where:
- A variable section depth is structurally more efficient, and
- Longer spans permit fabrication and materials cost savings.

Portions of haunched girders, such as cross frames and wind bracing, require special fabrication.

The use of steel box girders in the District is discouraged because of the difficulty they present in construction and maintenance. They are:

- Difficult to lift into place due to their size and weight
- Re-decking is more complex because of the need to maintain traffic and stage construction during deck removal and replacement (with composite design, the deck serves as an element of the compression flange)
- Inspection and maintenance of the interior of box girders is more difficult
- Complex geometric control is required for fabrication and construction

In selecting the type of stringers, the use of composite design with shear connectors on rolled beams without cover plates should be preferred.

In the design of welded plate girders, consideration should be given to minimizing the number of transverse intermediate stiffeners.

The use of transverse intermediate stiffeners is discouraged to the extent practical in exercising good judgment in design engineering practice for the following reasons:

- Elimination of projections and obstructions and the resulting flat surfaces optimize the chances of improved quality of workmanship in the cleaning and painting of the structural steel both in the fabricating shop, initial field coating and future maintenance painting.
- Fabricating cost differentials between welding stiffeners versus use of additional material in the main components of girders are not overwhelmingly significant and should be considered during design.

Consideration shall also be given to minimizing the number of butt-welded flange plate transitions. Plate size transitions may be located at the field splice so that butt-welding requirements are either reduced or eliminated. It is the Designer's responsibility to check the availability of plate sizes in order to determine the location of shop splices for flange plates.

Reduction of material weight is not necessarily the ultimate factor in determining span type selection. The bulk of the cost is in fabrication, delivery and erection.

Simplification and repetition of details, reduction of fabricating operations, and ease of erection are often better means of achieving minimum cost.

20.1.5 Fracture Control Plan

The construction specifications provide, "... steel bridge members or member components designated as Fracture Critical Members (FCM's) shall be subject to the provisions of the AASHTO Guide Specifications For Fracture Critical Non-Redundant Steel Bridge Members...".

Fracture critical members or member components (FCM's) are members or components of members whose failure would be expected to result in collapse of the bridge. The responsibility for determining if any bridge member or member component is in the FCM category, shall rest with the Structural Design Engineer. If it is determined that any member or member component is in the FCM category, the following note shall be shown on the structural steel plans:

<u>Fracture Critical Members</u>: Members or member components designated as FCM shall be subject to the provisions of the <u>1978 AASHTO Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members</u> (with current interims) and DDOT Amendments.

NOTE: Shop drawings shall be reviewed accordingly by the Structural Design Engineer.

20.1.6 Composite Design

Composite action decks are designed such that both the deck and beam or girder, respond to live loads and superimposed dead loads as a unit. Superimposed dead loads include all dead loads placed on the deck after it is cured. For steel beams, the interconnection is accomplished using studs attached to the top flange of the beam or girder.

Steel stringers with a concrete deck slab shall normally be designed as composite structures, assuming no temporary supports will be provided for the beams or girders during placement of the permanent dead load.

Shear connectors shall be M22 end welded studs. Height of studs depends on concrete haunch dimensions. Shear connectors shall penetrate at least 2 in. into the bottom mat of the deck slab, but the top of the stud head shall be 3 in. minimum below the top of the deck slab. Use of the same height stud on any one bridge is preferred. Stud-type shear connectors are used for both positive and negative moment areas. Studs with a 7/8 in. diameter are used. In negative moment areas, the maximum stud spacing is 24 in.

20.1.7 Camber

Beams must be cambered in the fabrication process. A camber diagram is needed for proper fabrication of the beam and must be included in the bridge plans. Camber deflections must be computed for each beam at the $1/10^{th}$ points of each span or at 10 ft. intervals, whichever is less; the same for finished deck elevations. The designer must furnish camber deflections for the following loadings:

- Dead load due to weight of structural steel,
- Dead load due to concrete deck,
- Dead load due to superimposed dead loads such as wearing surfaces, sidewalks, parapets, and utilities,
- Camber for vertical curve ordinate to meet proposed roadway profile, and
- Total of dead load deflections and camber.

In developing camber diagrams, the designer must consider the differences in loadings, such as the effects of sidewalks, parapets, and barriers, on individual beams and girders. The deflections caused by the dead load from the structural steel forms and reinforced concrete deck, are resisted by the steel superstructure. Deflections caused by superimposed dead load are resisted by the composite section comprised of the reinforced concrete deck and the beams and steel girders. The fascia beams likely will not deflect the same as interior beams. Consequently, a camber diagram must be provided for fascia beams as well as for interior beams.

Because the screed rail for the deck-finishing machine is set from the fascia beam, camber of the fascia beam is critical to achieve the correct deck elevations, the specified deck thickness and proper drainage. Because of the potential hazard from ponding and freezing of water on the deck, the designer must evaluate beam deflections, deck cross slope and roadway geometry as well as scupper locations to ensure that water drains properly.

20.1.7.1 Simple Spans

The various conditions of dead load deflection and camber for each simple span stringer shall be tabulated on the structural steel plans as shown below:

The column headed "Vertical Curve Ordinate" shall be used exclusively for simple span stringers located within the limits of a crest vertical curve. Where such stringers are located within the limits of a sag vertical curve, provision for its ordinates must be made within the concrete haunch. Consequently, the tabulation of its ordinates is unnecessary.

Total dead load camber is equal to the sum of the dead load deflections. An architectural camber shall be provided for all simple span stringers unless the vertical curve ordinate meets this, in which case the architectural camber may be omitted. The architectural camber has a parabolic curve with a vertical ordinate of L/1200 in the middle of the span, where L is the length of the span. When establishing the depth of the concrete slab and haunch in composite design, the following items shall be considered:

- Total camber required.
- Girder dimensional tolerances per Section 3.5 of the ANSI/AASHTO/AWS Bridge Welding Code D1.5.
- A minimum cover of 3 in. over the shear connectors.

When total camber is less than minimum that can be maintained in a beam (W Section) no camber is required but a note stating, "Beams shall be placed with any mill camber up" shall be shown on the drawings.

20.1.7.2 Continuous and Cantilevered Spans

The various conditions of dead load deflections and cambers for each stringer shall be tabulated at the tenth point of spans and at the field splice points (at dead load points of contraflexure if field splices are not provided).

The following table shows an example of a typical tabulation for a continuous span.

Table 20-A

CAMBER TABLE																																			
	Centerline Brgs. Abut	SPAN 1										**Centerline Brgs. Pier 1**	SPAN 2											**Centerline Brgs. Pier 2**	SPAN 3										**Centerline Brgs. Abut**
POINT NUMBER		1	2	3	4	5	6	7	IP1	8	9	10	11	12	IP2	13	14	15	16	17	IP3	18	19	20	21	22	IP4	23	24	25	26	27	28	29	
Steel	0											0												0											0
Conc. Slab	0											0												0											0
Forms and Added Concrete Thickness	0											0												0											0
S. D. L.	0											0												0											0
V. C.	0																																		0
Architec-tural	0											0												0											0
TOTAL	0											0												0											0

20.1.7.3 Camber Table Notes

- The total camber as tabulated is assumed to be measured vertically to the top of the fully cambered web from a straight line drawn from the intersection of top of web and centerline of bearing at one end of the girder to the intersection of top of web and centerline of bearing at the other end of the girder.
- The camber labeled “Steel” in the table is the camber required in the girder to offset the deflection due to the dead load of the steel in the girder.
- The camber labeled “Conc. Slab” in the table is the camber required in the girder to offset the deflection due to the dead load of the concrete slab.
- The camber labeled “S.D.L.” in the table is the camber required in the girder to offset the deflection due to the superimposed dead load, that is, the curb, sidewalk, railing and future, wearing surface.
- The camber labeled “ Forms and Added Concrete Thickness” is the camber required in the girder to offset the deflection due to the weight of the forms and due to the weight of added concrete that is needed to meet the deck grades.
- The camber labeled “V.C.” in the table is the camber required in the girder to follow the vertical curve. The Vertical Curve value shall be used exclusively for stringers located within the limits of a crest vertical curve. Where such stringers are located within the limits of a sag vertical curve, provision for its value must be made within the concrete haunch. Consequently, the tabulation of its values is unnecessary.
- The camber labeled “Architectural Camber” shall be calculated as a parabola with a vertical middle ordinate of L/1200, where “L” is the span length. If the vertical curve value provides this camber value, the architectural camber may be omitted.
- Cambers listed in the table as positive are upward cambers.
- Cambers listed in the tables as negative are downward cambers.
- The cambers are tabulated in inches.

20.1.7.4 Sag Cambers

Because of the objectionable appearance of a sag camber in a stringer, sag or negative cambers must be avoided. The following are a few guidelines on possible means of avoiding negative camber in a stringer:

- Avoid sag vertical curves on bridges.
- Never begin or end a superelevation transition or runoff in the middle of a span. Always begin or end transitions off the structure or, if this is impossible, begin or end the transition at a centerline of bearing or a centerline of pier.
- Never place a sag camber in a straight stringer on a curved roadway in order to accommodate the variation in the theoretical bottom of slab elevation. The variation should be taken up in the haunch.
- Upward dead load deflection may occur in some areas of continuous girders when the ratio of maximum to minimum span lengths becomes significant. There always is a possibility that computed camber built into the girder is not completely removed with the application of dead load. Camber due to a future wearing surface will remain when construction is completed. Additional camber may remain due to differences between design assumptions and actual girder performance.

20.1.8 Multiple Span Structures

It is desirable, from an aesthetic viewpoint that a uniform depth of concrete fascia is kept for the full length of the exposed fascia. All fascia beams shall be set so that the bottom of the top flanges will be aligned.

Stringers, beams, and girders shall generally be of uniform depth for the full length of the structure, except where changes in depth are absolutely necessary to meet underclearance requirements or where a change in depth is desirable to enhance the appearance of the structure. Changes in depth shall not normally be made in structures with varying spans. Interior stringers shall be made the same depth as the fascia stringer.

20.1.9 Diaphragms and X-Frames

Diaphragms, cross frames, and lateral bracing are used to stiffen and connect beams so they work as a unit. Diaphragms are used to connect rolled beams with 36 in. height or less. Channel diaphragms for rolled beams shall be at least one-half the beam depth. Cross frames are used for rolled beams greater than 36 in. high and plate girders. Cross frames shall be at least 3/4 of the girder depth. Refer to details that were adapted from standards developed by the FHWA Region 3 Structural Committee for Economical Fabrication.

End diaphragms and their connections shall be designed for the effect of wheel loads which they may be required to support, for the effect of transverse movement of the bearing shoes due to temperature differences between the superstructure and substructure, and for the effect of all horizontal superstructure forces. The diaphragms and their connections shall be designed for the transverse force necessary to move the bearing shoes, in appropriate combinations with the other forces listed above.

NOTE: DDOT also has standard diaphragms.

For severely skewed (60 degrees ±) structures, the structural steel layout should be examined to determine if the location of relatively stiff intermediate diaphragms placed normal to the stringers introduce detrimental stresses in diaphragms and stringers due to twisting. If the condition exists, consider staggering the spacing of the diaphragms or adding the following note:

"Intermediate diaphragm connections to stringers shall be limited to finger-tight bolts in oversized holes until the dead loads are in place. The bolts shall be tightened after the deck is in place."

Transverse stiffeners may be either intermediate or bearing. For girders with webs of 54 in. or smaller, it is preferable not to use intermediate stiffeners. For girders with webs larger than 54 in., the web thickness may be increased to limit the transverse stiffeners to only one or two locations per span beyond those provided for diaphragm or cross frame connections. Transverse stiffeners must be a minimum of 3/8 in. in thickness. Stiffeners shall be welded to the web with a minimum ¼ in. continuous fillet weld. Intermediate stiffeners will be welded to the compression flange and tight fit to the tension flange. Bearing stiffeners shall be welded to both the top and bottom flanges. Transverse stiffeners used as connection plates for diaphragms will be welded or bolted to both flanges, and the flange stress shall be investigated for fatigue. Transverse stiffeners will be clipped as shown on the plans.

Longitudinal stiffeners are used to improve the bending resistance of welded-plate girders. Because of the increased fabrication complexity and greater likelihood of occurrence of welds or weld-intersection flaws, longitudinal stiffeners may be used only with the approval of the Bridge Design Engineer. The longitudinal stiffeners should always be placed on the opposite side of the web from the transverse intermediate stiffeners to minimize the number of intersections between longitudinal and transverse stiffeners. Transverse intermediate stiffeners used for diaphragm connection plates must be placed on both sides of interior beams. Longitudinal stiffeners may not be continuous and may be cut at their intersections with transverse stiffeners. Close attention is needed to the details at the intersection of longitudinal and transverse stiffeners. Avoid intersecting welds, if possible, by stopping the welds short of the intersection. Where intersecting welds cannot be avoided, nondestructive testing (NDT) must be specified to detect weld flaws that may cause cracking.

20.1.10 Stability Between Transportation and Erection

The stability of the stringers and girders during transport and erection is normally the responsibility of the Contractor, but, wherever possible, the design should be such that temporary bracing or diaphragms are not required. In reviewing shop drawings, Engineers shall satisfy themselves that the Contractor has properly met his contractual responsibilities in this respect.

20.1.11 Welded Details

Field Welding to stringers, plate girders or any major component of the structure shall not be permitted.

Field welding, when it is deemed absolutely necessary for the minor bridge elements, shall conform to the following Sections of ANSI/AASHTO/AWS Bridge Welding Code D1.5. The following parameters shall be included in the Special Provisions:

- Pre-qualification of the proposed welding procedures shall be in accordance with Section 5, Part A.
- Qualifications of the welding operator shall be in accordance with Section 7, Part B.
- The Quality Control Inspector shall meet the qualifications specified in Section 6.1.3 and 12.16.
- All fillet welds shall be 100 percent Magnetic Particle (MT) tested in addition to Visual Inspection.

The **ANSI/AASHTO/AWS Bridge Welding Code D1.5** promulgates the following concepts of inspection, which, in effect, are separate functions:

- Fabrication/Erection Inspection and Testing (Quality Control) is to be performed by the Contractor or Fabricator as a mandatory requirement.
- Verification Inspection and Testing (Quality Assurance) is the prerogative of DDOT.

The Department follows the **ANSI/AASHTO/ AWS D1.5-88 Bridge Welding Code** for welding design. Electro-slag welding is one exception to the Code; its use is not permitted for the District bridges. Fillet welds are preferred over other types of welds because they are easier to make with automated welding equipment.

Provisions in the **ANSI/AASHTO/AWS Bridge Welding Code D1.5** requires that contract documents identify main members and also that contract documents identify groove welds in these members as to category of stress (tension, compression or reversals of stress). Both of these identifications are needed to define the extent of non-destructive testing required by the Contractor as a minimum level under QC inspection specifications. Identification of the nondestructive inspection required for all welds included in **Section 6.7, Parts B and C, of the ANSI/AASHTO/AWS Bridge Welding Code D1.5**, shall be accomplished by providing symbols and notes.

For main member components in structure types such as trusses, bents, towers, box girders etc., it shall be the Structural Design Engineer's responsibility to identify such members and welds as part of the details on the contract drawings with the appropriate welding and NDT symbols.

Certain miscellaneous details (supports for screed rails, steel deck forms, miscellaneous connection plates, gussets, etc.) shall normally not be welded by the use of fillet welds (regardless of the direction of weld), plug welds, or tack welds to members or parts subject to tensile stress. At locations where welding cannot be avoided, the maximum stress at the point of attachment shall not exceed the allowable fatigue stress range, F_{sr}, computed from the **AASHTO Specification, Table 10.3.1A, Category F.**

The attachment of these details shall not be allowed where the stress exceeds F_{sr}.

The contract plans and shop drawings shall clearly show the flange areas where no welding is permitted and the areas on continuous girders where the stiffeners are to be connected to the top or bottom flanges.

20.1.12 Field Splices

To facilitate the fabrication, shipping, and the erection of steel girders, one optional field splice will be permitted in spans between 115 and 150 ft. in length. This field splice shall be located between the 1/3 and outer 1/4 points of the span length. When the span exceeds 150 ft., optional field splices may be located between each of the 1/3 and outer 1/4 points. In continuous spans, the bolted field splice shall preferably be made at or near the points of dead load contraflexure. Locations and details of the optional field splice shall be shown on the plans. The Contractor may request modifications subject to approval by the Engineer.

Field splices shall be designed and detailed using AASHTO M164 (ASTM A 325) high strength bolts. The flanges should have sufficient excess area at points where splicing is anticipated to permit a bolted splice to be made. All field splices of beams or girders and connections will be bolted. Construction considerations include site conditions and weight of the beam.

Normally, 7/8 in. diameter, M164M bolts are specified for field connections. In some circumstances, higher strength M253M bolts may be considered to avoid an excessive number of bolts in splices or connections. All bolts in a bridge should be the same diameter. Avoid bolts over 1 in. in diameter.

The Department commonly uses two types of high-strength bolts: Type 1 and Type 3. Type 1 bolts are made from medium carbon steel. Type 3 bolts have atmospheric corrosion and weathering-resistant characteristics comparable to weathering steel. Type 1 M164M bolts are painted after installation. Hot-dip galvanizing is not permitted.

Splice plate thickness should be increased where longitudinal web stiffeners are terminated at bolted field splices. Thicker webs can be used to eliminate the need for longitudinal stiffeners.

Splice locations are generally selected near transitions in flange thickness or width where there is sufficient flange area to permit hole drilling while still maintaining the required net area.

When rolled beams are used for continuous structures, the field splices should be located in areas where no cover plates are required. Consideration should be given to the fact that the fatigue strength of the section adjacent to the bolted connection (Category B*) is less than the fatigue strength of the base metal in areas where there is no splice (Category A*). *See **Article 10.3 of the AASHTO Standard Specifications for Highway Bridges.**